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1. REPORT DATE (DD-MM-YYYY) 2/Jan/2002	2. REPORT TYPE MAJOR REPORT	3. DATES COVERED (From - To)		
4. TITLE AND SUBTITLE HYDRAULIC CONDUCTIVITY AND DRAINED STRENGTH OF RECYCLED ASPHALT PAVEMENT AND CRUSHED CONCRETE		5a. CONTRACT NUMBER 5b. GRANT NUMBER 5c. PROGRAM ELEMENT NUMBER 5d. PROJECT NUMBER 5e. TASK NUMBER 5f. WORK UNIT NUMBER		
6. AUTHOR(S) CAPT CARLEY PATRICK J				
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) UNIVERSITY OF TEXAS AUSTIN		8. PERFORMING ORGANIZATION REPORT NUMBER CI01-314		
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) THE DEPARTMENT OF THE AIR FORCE AFIT/CIA, BLDG 125 2950 P STREET WPAFB OH 45433		10. SPONSOR/MONITOR'S ACRONYM(S) 11. SPONSOR/MONITOR'S REPORT NUMBER(S)		
12. DISTRIBUTION/AVAILABILITY STATEMENT Unlimited distribution In Accordance With AFI 35-205/AFIT Sup 1		<b>DISTRIBUTION STATEMENT A:</b> Approved for Public Release - Distribution Unlimited		
13. SUPPLEMENTARY NOTES				
14. ABSTRACT				
<b>20020204 076</b>				
15. SUBJECT TERMS				
16. SECURITY CLASSIFICATION OF: a. REPORT   b. ABSTRACT   c. THIS PAGE		17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES 59	19a. NAME OF RESPONSIBLE PERSON  19b. TELEPHONE NUMBER (Include area code)

## **Abstract**

# **Hydraulic Conductivity and Drained Strength of Recycled Asphalt Pavement and Crushed Concrete**

by

Patrick James Carley, Captain, USAF

The University of Texas at Austin, 2001

Master of Science in Engineering

Report Length: 59 Pages

Recycled asphalt pavement (RAP) and crushed concrete (CC) are potentially good fill materials for many highway construction projects. The Texas Department of Transportation is considering the suitability of using RAP and CC as backfill in the construction of mechanically stabilized earth (MSE) walls. This report focuses on the laboratory measurement of the geotechnical properties of these materials. Tests were conducted on 4-inch (100 mm) diameter triaxial specimens of compacted RAP and CC to measure the hydraulic conductivity and drained shear strength of each material. Comparison tests were also performed on a more conventional crushed limestone fill material. Based on these tests, RAP appears to exhibit adequate strength and hydraulic conductivity for use as an MSE wall backfill. However, additional tests are warranted to study possible creep strength loss in RAP. CC also exhibits adequate strength, but has a relatively low hydraulic conductivity. Additional tests are warranted to determine how the hydraulic conductivity of CC is affected by the finer fraction of this material.

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Recycled Asphalt Pavement and Crushed Concrete**

**by**

**Patrick James Carley, B.S.**

**Report**

Presented to the Faculty of the Graduate School of  
The University of Texas at Austin  
in Partial Fulfillment  
of the Requirements  
for the Degree of

**Master of Science in Engineering**

**The University of Texas at Austin**  
**December 2001**

**Hydraulic Conductivity and Drained Strength of  
Recycled Asphalt Pavement and Crushed Concrete**

Approved by  
Supervising Committee:

Ellen M. Raths  
Alan Rauch

## **Acknowledgements**

I would like to thank my advisor and co-supervisor, Alan F. Rauch, for contributing his time, knowledge, insight, and positive attitude. Dr. Rauch always had time for the most important aspect of a university, the students. I also would like to thank my co-supervisor, Ellen M. Rathje, who counseled me with my research and provided guidance on the formulation of this report. Additionally, she was an inspiration in the gym each morning at 6:00am. I also would like to acknowledge the four other faculty members of the geotechnical section who have contributed to my education.

I also would like to thank the United States Air Force for making my graduate education a possibility. A special thanks to Chirayus (Ake) Viyanant. Ake has put in countless hours to this project and will be conducting related research through the entire project duration. Good luck Ake.

Last, but not least, to my wife Stephanie and my son Joshua who was born during my time here at UT. Thanks for the never-ending encouragement and warm smiles and hugs after I'd return home from a day of classes or research. You both make it easy to remember what the most important things in life are.

November 2001

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Patrick James Carley, M.S.E.

The University of Texas at Austin, 2001

Co-Supervisors: Alan F. Rauch and Ellen M. Rathje

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## **Chapter 1: Introduction**

### **1.1 BACKGROUND**

The value of aggregates produced in the United States exceeded \$7.7 billion in 1991. This was the largest non-fuel mineral commodity produced (Chini et al. 1996). Approximately half of this production went to building construction, with the remainder going to public works (Poulin et al. 1994).

With the growing concern for environmental issues in this country and in various industrialized countries around the globe, increased political and ecological opposition to the production of sand and gravel by dredging huge cavities in the traditional landscape are ever present (Figure 1.1). Further exploitation of natural resources of this kind is no longer acceptable because of increased urbanization and growing public concerns with environmental issues.

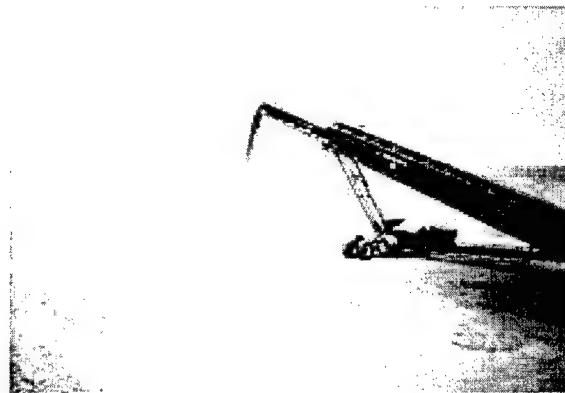


Figure 1.1. Quarry Operations (Pit and Quarry 2001).

In conjunction with the decreasing availability of virgin aggregate sources, there is a need to reduce the waste materials discarded in landfills, including construction debris and building product wastes. In urban areas where landfill space is scarce and dumping concrete and asphalt pavements can be costly, crushing old concrete and asphalt pavements and reusing them as aggregate is cost effective and eliminates disposal problems and tipping fees. Recycled aggregates may be acceptable for road base materials, hot mix asphalt (HMA), and portland cement concrete (PCC) pavements.

Natural aggregates and landfill space are not infinite. With its high population density and limited amount of land available for landfill sites, Japan reused 48% of its concrete rubble associated with demolition work in 1990 (Kasai 1994). In the United Kingdom, 40% of its demolition waste is recycled, mainly for low grade applications such as fill and hardcore (Collins 1994). Gradually, public agencies and private companies in the United States are discovering it is economically beneficial to use recycled materials in pavement construction. In the New York City metropolitan area, recycled PCC is almost exclusively the material of choice (Snyder 1996). In Maine, the availability of natural aggregates is in jeopardy because many sources for gravel in the state are highly radioactive (Redmond 1996). This may force Maine to use more recycled aggregates.

On the national level, the 1997 Department of Transportation Appropriations Bill included a section to study applications in which significant volumes of secondary waste can be used while ensuring long-term effectiveness and performance and without increasing construction costs. One of the research

areas supported by the appropriations bill led to the development of a method to evaluate the potential of a waste material to be reused (FHWA 2001a). Since 1981, the Federal Highway Administration (FHWA) policy on pavement recycling has been clear and to the point; the policy states that recycled pavement should be allowed for use on all projects.

## **1.2 PROJECT GOALS AND OBJECTIVES**

The University of Texas at Austin and Texas A&M University, with the support from the Texas Department of Transportation, are evaluating the use of recycled asphalt pavement (RAP) and crushed concrete (CC) as backfill for mechanically stabilized earth (MSE) retaining structures. Olson (2000) describes MSE structures as those having exposed facing material supported by tensile reinforcement buried in the backfill.

This research project (TxDOT No. 0-4177) has two primary objectives: the evaluation of the geotechnical properties of RAP and CC backfill and the assessment of reinforcement durability when buried in these materials. This information is needed to assess if durable and reliable MSE walls can be designed and constructed with RAP and CC backfills. The project will consist of an extensive laboratory investigation that will fully characterize RAP and CC and evaluate the effect of these materials on the durability of metallic and geosynthetic reinforcement typically used in MSE walls.

This report concentrates on measurements of the hydraulic conductivity and shear strength of RAP, CC, and, as a comparison, a conventional fill material.

Tests were conducted in a triaxial cell on 4-inch (101.6 mm) diameter, compacted specimens of each material.

## **Chapter 2: Crushed Concrete and Recycled Asphalt**

### **2.1 PAVEMENTS**

Historically, pavements have been divided into two broad categories: rigid and flexible. These classical definitions, in some ways, are an oversimplification. However, the terms rigid and flexible provide a good description of how the pavements react to loads and the environment.

Rigid pavements are made up of portland cement concrete and may or may not have a base course between the pavement and subgrade. Flexible pavement is an asphalt pavement. It generally consists of a thin asphalt wearing surface built over multiple layers of base and subbase courses, which are usually gravel or stone. These layers rest upon a compacted subgrade or compacted soil.

The basic difference between the two types of pavements is the manner in which they carry surface traffic loads. Because concrete is relatively rigid, a rigid pavement contains most of the structural capacity in the concrete layer. Additionally, the rigid pavement tends to distribute traffic load over a wide area of the subgrade. On the other hand, flexible pavements are structurally weaker. Therefore, traffic loads are spread over a smaller area of the subgrade, which explains the need for multiple layers and greater overall roadbed thickness.

Concrete pavement is basically a mixture of two components: aggregates and paste. Paste consists of cement, supplementary cement materials, and water. Aggregates make up about 60% to 75% of the total volume of concrete (ACPA 2001). Asphalt pavement consists of aggregates and a bitumen component.

Aggregates comprise an even larger percentage of the total volume of material than for concrete pavement, usually 85%. As noted previously, aggregate production is the largest non-fuel commodity. Reducing the use of virgin aggregate by recycling pavements will not only prove economically beneficial, it will also conserve our natural resources.

## **2.2 PERFORMANCE OF RECYCLED PAVEMENTS**

In the early 1980s, Halverson (1983) conducted a research project with the Minnesota Department of Transportation that evaluated the performance of a rigid pavement constructed with recycled crushed concrete aggregate. A 16-mile section of US-59 in southwestern Minnesota was selected. Upon completion of the roadway, a roughometer test was conducted, which was used to quantify ride quality by measuring the roughness of the pavement surface. Any score of less than 85 inches/mile of roughness was deemed acceptable. The average roughness for the 16-mile section was 71 inches/mile. Additionally, the contractor was awarded a bonus for constructing 41% of the total lane miles with a roughness less than 69 inches/mile.

On the same project, an economic analysis was performed to compare the actual cost of the project using recycled crushed concrete aggregate versus the cost of a traditional concrete pavement. The results indicated a savings of over \$700,000 using the recycled material. The authors noted that the scarcity of quality virgin aggregate sources in the area was partly responsible for the large financial savings.

Other projects have also demonstrated the successful use of recycled crushed concrete in rigid pavements. Oklahoma became the first state to recycle a full-size project on an Interstate Highway in the early 1980s. Michigan followed in 1983 with a project that replaced 5.7 miles of four-lane divided highway with a pavement utilizing recycled crushed concrete aggregate. Bid prices were so good on the first project that they advertised a second project immediately following completion of the first (Hansen 1992).

Similar to using recycled crushed concrete in rigid pavement, recycled asphalt has proven to be a successful material in new pavements. The Center for Transportation Research at the University of Texas issued the following statement (Kennedy and Anagnos 1983):

Based on the results of this study and previous studies it was concluded that the engineering properties of hot mixed recycled asphalt mixtures are comparable to those of conventional mixtures although there is a tendency for the recycled mixtures to be more brittle. Thus the amount of added recycling agent, commercially available softening agents, or soft asphalts, should theoretically be sufficient to soften the reclaimed asphalt to a level equal to that of normally used virgin asphalt.

### **2.3 GEOTECHNICAL USES OF RECYCLED ASPHALT PAVEMENTS**

In addition to recycling crushed pavements into new paving materials, some recycled asphalt pavements have been used for embankment construction or fill material. Although not extensive, the use of RAP as an embankment construction material has been reported in at least nine states (Ahmed 1991).

Some of the engineering properties of RAP that are of particular interest when used in embankment applications include gradation, compacted density,

moisture content, shear strength, consolidation characteristics, hydraulic conductivity, durability, and corrosivity. The design procedures for embankments or fill containing RAP are the same as design procedures for conventional embankment materials. The design should take into consideration slope stability, settlement or consolidation, and bearing capacity concerns.

The same methods and equipment for compacting conventional fill can be used for compacting crushed RAP or blends of soil and RAP. Where large, broken pieces of old asphalt pavement are incorporated during embankment construction, additional attention is needed during compaction to ensure that no large voids are formed within the fill that could contribute to subsequent long-term differential settlement. Quality control procedures should be applied in the same manner as for conventional fill. Visual inspections on a continuous basis should be performed to ensure that the specified degree of compaction is achieved, and that there is no movement under the action of compaction equipment.

The Federal Highway Administration (FHWA 2001a) indicates that at least five states have used RAP directly as a backfill material. Additional states have used RAP as an additive in embankment construction. In each case, the performance of RAP backfill was generally considered as satisfactory to good. The required construction procedures for a RAP embankment are generally the same as the procedures used for conventional embankments. However, FHWA has issued recommendations regarding construction procedures for RAP embankments:

- 1) Random sampling and testing of the RAP stockpile must be performed because various sources of RAP may be different.
- 2) Additional attention must take place during compaction to ensure that no poorly compacted zones are created in the fill, leading to long-term differential settlement.
- 3) Some jurisdictions may require a minimum separation distance between water sources and fill materials containing RAP to avoid submersion of RAP in water, because water leaching from RAP may be a potential environmental concern.

## **2.4 GEOTECHNICAL USES OF CRUSHED CONCRETE**

Although the use of crushed concrete in embankments or fill may not make the best use of this high quality aggregate, in areas lacking good fill material CC can be used for this application. Many specifying agencies consider CC aggregates to be conventional aggregate. Other agencies require only minimal processing to satisfy the conventional soil and aggregate physical requirements for embankment or fill material (FHWA 2001b).

Most states permit the use of crushed concrete for varying applications. Chini (1996) sent out questionnaires to the materials engineers at all 50 state departments of transportation. Receiving 40 responses, 32 indicated the use of recycled concrete was allowed by their state agencies. Table 2.1 lists the various applications associated with the use of recycled PCC. The results show that

recycled CC is primarily used in non-structural areas. This includes applications such as base course, subbase, backfill, and rip-rap.

Table 2.1. Reclaimed PCC Aggregate Applications (Chini 1996).

<b>APPLICATION</b>	<b>NUMBER OF STATES PERMITTING USE OF RECYCLED PCC*</b>
BASE COURSE	21
PCC PAVEMENT	10
ASPHALT PAVEMENT	10
BACKFILL	7
SUBBASE	6
RIP-RAP	4
MISCELLANEOUS CONCRETE	3
MECHANICAL SOIL STABILIZATION	1
CONCRETE REHABILITATION	1
NOT ALLOWED IN PRE-STRESSED	1

\*out of 40 responding to the questionnaire

The Texas Department of Transportation (TxDOT 1999) has reported three projects that used CC as backfill material. The Corpus Christi, Lufkin, and Beaumont Districts all reported excellent to good results with their respective projects involving CC backfill material. These projects spanned a period from 1977 to 1994.

Three major concerns when using CC as a backfill material in MSE walls are the potential corrosion of metallic reinforcement, low hydraulic conductivity of the material, and caking of the material while stockpiling prior to use. Corrosion of the metallic reinforcement is based on the hypothesis that the high pH of a crushed concrete-water mixture will increase the rate of steel corrosion. Popova et al. (1998) studied the corrosive behavior of CC in MSE walls. He found the rates of corrosion for a galvanized steel rod embedded in the fill material was the same for both CC and granular soil fill at the beginning of the test (approximately 0.02 mm/year). However, the rate of corrosion increased with time for the CC material (0.075 mm/year at 400 days), while it decreased for the granular soil fill (0.005 mm/year at 400 days).

Another concern is that the hydraulic conductivity of crushed concrete may be too low to act as a free draining material. Hansen (1992) discusses various concerns with using crushed concrete as a drainage material:

- 1) Recycled fines should not be used in drainage layers. “Fines” is defined by Hansen (1992) as the minus No. 4 material (4.75 mm). Some of the cementitious material attached to the surfaces of the fines goes into solution when water percolates through the material. A precipitate then

forms in the drainage structure. Therefore, Michigan DOT no longer allows the use of recycled fines in drainage layers. (Note that the specimens tested in the present research project had approximately 35% by weight smaller than 4.75 mm).

- 2) Hydration of cement paste can potentially lead to low values of hydraulic conductivity. Approximately 30% by volume of the 16-32 mm size recycled aggregate was old mortar in the Hansen (1992) study.

Corresponding figures were 40% of the 8-16 mm fraction and 60% of the 4-8 mm fraction. Fine recycled aggregate below 4 mm contained approximately 20% by weight old cement paste, while filler fraction (0.0-0.3 mm) may contain as much as 65% old cement paste.

These statements by Hansen indicate the potential for crushed concrete to exhibit some properties of freshly mixed concrete when water is introduced.

Excessive fine material in CC may provide an explanation to the values listed by Carter and Bentley (1991) concerning the hydraulic conductivity of various highway materials. They indicate that concrete sand with a low dust content can have hydraulic conductivity values between  $7.0 \times 10^{-2}$  and  $7.0 \times 10^{-4}$  cm/sec. Additionally, they state that concrete sand with a high dust content can have values between  $7.0 \times 10^{-4}$  and  $7.0 \times 10^{-6}$  cm/sec. Unfortunately, they do not define "low dust" or "high dust". Additionally, they do not provide values for CC. According to Carter and Bentley (1991), any value below  $1.0 \times 10^{-4}$  cm/sec is considered a poor drainage material.

The physical phenomenon of caking (i.e., the conglomeration of particles), although not a significant factor for strength or compaction, may potentially lead to changes in hydraulic conductivity. Smaller particles, which impede the flow of water, may become cemented together to form larger particles in the material stockpile, prior to compaction. Effectively reducing the amount of fines, this could yield higher hydraulic conductivities. Conversely, continued hydration in the embankment may close off some pore spaces, thereby impeding seepage and yielding a lower hydraulic conductivity. Hansen (1992) indicates crusher fines below 4 mm may contain up to 4% by weight calcium hydroxide, which is formed by hydration of the original cement in the old concrete. When mixed with water and left to dry in the laboratory, the product will gradually harden, much like a weak lime mortar, and may give rise to caking in stockpiles. Caking was observed in oven-dried samples of CC during this research project.

## **Chapter 3: Testing Procedures**

### **3.1 INTRODUCTION**

The objective of this project is to evaluate the geotechnical properties of CC and RAP as backfill materials. This information is needed to assess if MSE walls can be adequately designed and constructed with CC and RAP.

This report provides the results and observations from laboratory testing of CC, RAP, and a conventional fill material (CFM). The conventional fill material was tested to provide a basis of comparison for the data obtained on CC and RAP. Results from a total of 13 consolidated-drained triaxial shear tests are included in this report. Four tests were performed on CC and CFM specimens and five were conducted on RAP. The effective confining pressures ranged from 12 to 45 psi (82.7 to 310.3 kPa). Prior to each respective triaxial test, the hydraulic conductivity of each specimen was measured. The specific gravity of the test materials was 2.66 for the CFM, 2.62 for CC, and 2.28 for RAP. These values were used for all test computations.

All hydraulic conductivity ( $k$ ) and consolidated-drained (CD) triaxial tests were performed in the geotechnical testing laboratory in Ernest Cockrell Jr. Hall on the campus of the University of Texas at Austin. The equipment used for this research was manufactured by Geotechnical Consulting and Testing Systems (GCTS) of Tempe, Arizona.

### **3.2 TESTING EQUIPMENT**

The triaxial testing apparatus used for these tests is a versatile system that may be used to perform a variety of geotechnical tests including monotonic triaxial tests, cyclic triaxial tests, hydraulic conductivity tests, and resilient moduli tests. Specimens as large as 4-in (101.6 mm) diameter can be tested. The system uses three closed-loop, digital, servo valves to control the axial load (hydraulic), cell pressure (pneumatic), and pore pressure (pneumatic). The system was designed to comply with all specifications of ASTM D5311-92 for cyclic triaxial strength testing. The cell pressure and back pressure may be operated manually or with computer-controlled servo valves. The volume change panel provides 250 ml of capacity with a resolution as small as 0.1 ml. The GCTS apparatus is operated by a microcomputer using a Windows-based program named TEST. Within TEST, there are different modules that run the various geotechnical tests. A photograph of the testing apparatus is shown in Figure 3.1.

### **3.3 SPECIMEN PREPARATION**

For the purposes of this research, a reference gradation curve was generated to represent all three source materials. All test specimens of RAP, CC, and CFM were mixed in proportions to match this reference gradation. However, the triaxial specimens were limited to 4-in (101.6 mm) diameter. For this specimen size, a maximum particle size of 0.67 in (17 mm) is allowed, which corresponds to 1/6 of the specimen diameter. Because of this limitation, particles

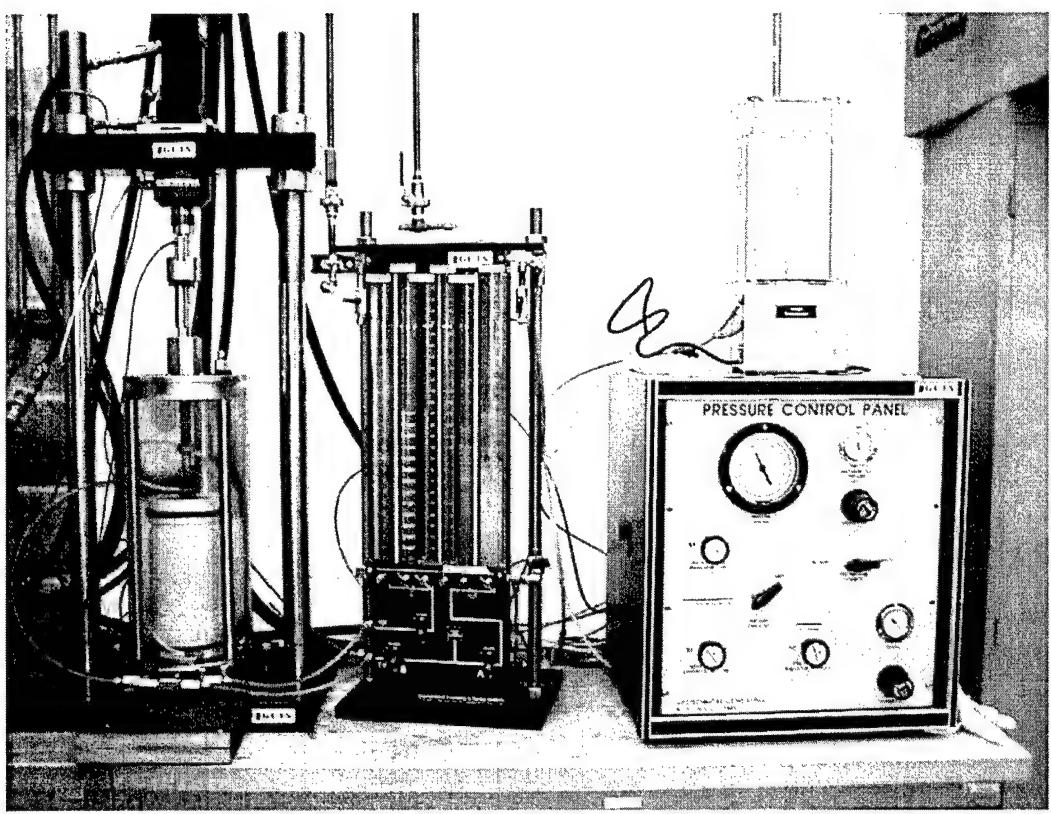


Figure 3.1. Triaxial Test Equipment.

larger than 0.67 in (17 mm) were removed. A 0.63 in (16 mm) sieve was used to scalp out the larger particles. Figure 3.2 shows the reference material gradation along with the modified gradation used to form the 4-inch diameter test specimens for triaxial testing.

A steel compaction mold was purchased to form the specimens. The mold has a 4-in (101.6 mm) inside diameter and is 8 in (203.2 mm) high. The mold is split on one side to facilitate the removal of compacted specimens. A 3 in (76.2 mm) high collar was attached to the top of the mold to properly compact the entire specimen.

The compaction effort, defined by Texas Department of Transportation specification Tex-113-E, was used in forming the test specimens. This procedure requires a compaction effort (C.E.) of 13.26 foot-pounds per cubic inch (ft-lb/in<sup>3</sup>). The compaction effort is calculated as:

$$C.E. = \frac{Ht. \text{ of drop (ft)} * Wt. \text{ of hammer (lbs)} * \# \text{ of drops} * \# \text{ of layers}}{\text{Volume of mold (in}^3\text{)}} \quad (3.1)$$

Here, the test specimens were compacted using a 10-pound (4.5 kg) hammer with a 1.5 foot (457.2 mm) drop height. The mold used resulted in a specimen volume of 100 in<sup>3</sup> (1.65\*10<sup>-3</sup> m<sup>3</sup>). The specimens were compacted in 8 equal layers. Therefore, 11 drops per layer were used to impart a compactive effort of 13.13 ft-lb/in<sup>3</sup>, which is consistent with the requirements of Tex-113-E.

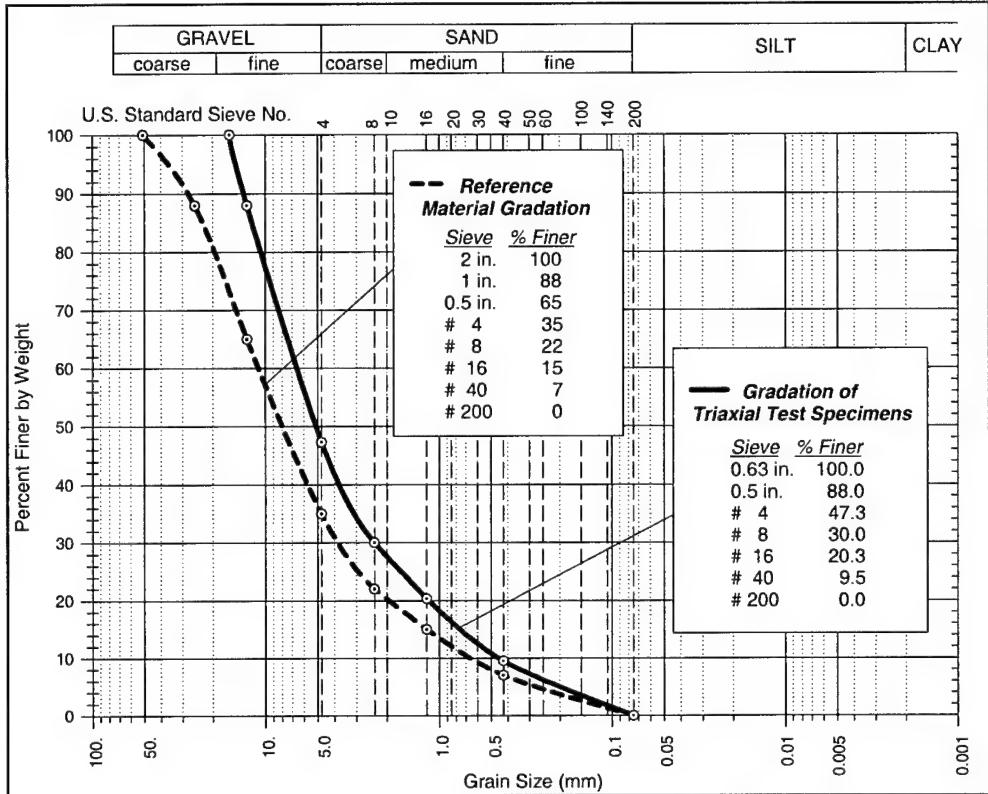


Figure 3.2. Reference Material and Triaxial Test Specimen Gradations.

Following compaction, the collar was removed and the specimen was trimmed to the proper height of 8 inches. The specimen, still in the mold, was then placed on a scale to measure its total weight.

Initial (after compaction) and final (after triaxial testing) water contents were measured by oven drying portions of the specimens (Figure 3.3). Prior to triaxial testing, water contents were measured by drying specimens at 60°C and 110°C, to see if any materials was ignited and lost at higher temperatures. This



Figure 3.3. RAP Specimen Removed From the Drying Oven.

was a potential concern for the RAP specimens. However, both temperatures produced similar water contents. Therefore, the 110°C oven was used for all water content measurements.

Specimens were prepared at approximate water contents of 9% for the CFM, 14% for the CC, and 4% for the RAP. Actual water contents of the specimens are provided in Chapter 5.

### **3.4 ASSEMBLY OF TEST APPARATUS**

The specimens were removed from the mold by turning two bolts on the mold to spread the split and increase the diameter. All three materials maintained their shape with no confining pressure applied. The specimens were then placed on the bottom platen of the triaxial cell with a porous stone and filter paper between the specimen and the platen. Figure 3.4 depicts a conventional fill material specimen placed on the bottom platen, porous stone, and filter paper.

The top platen, with a rubber membrane (0.025 inches thick, 4-inch diameter, 11 inches long) already rolled on, was placed on top of the specimen. The membrane was then carefully rolled down the specimen. After the membrane surrounded the entire specimen, two O-rings, which were pre-positioned around the bottom platen, were brought in place over the membrane to provide a seal at the bottom platen. Next, the top platen was removed and the integrity at the top of the specimen was checked. If a small portion of the material became dislodged during the membrane installation phase, extra material from the specimen preparation bin was inserted and compacted with finger pressure to maintain a

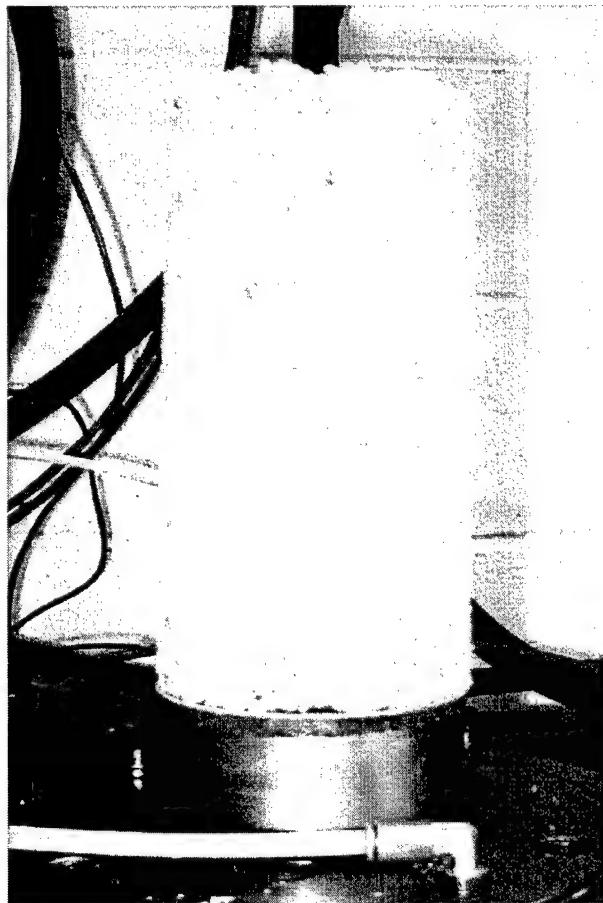


Figure 3.4. Conventional Fill Material on Bottom Platen.

nearly uniform 4-in (101.6 mm) diameter and 8 in (203.2 mm) height. Once completed, a piece of filter paper and a porous stone were inserted in the top opening of the membrane. The top end platen was then placed on the porous stone. The height of the specimen was checked in four locations around the specimen. This measurement provided the initial volume and, along with the weight, provided the total unit weight of the specimen. Finally, the top two O-rings were placed around the membrane on the top end platen. A final visual check of the membrane integrity and the placement of the four O-rings was performed prior to cell assembly.

Next, the cell was assembled and water was introduced into the cell. A small cell pressure of approximately 5 psi (34.5 kPa) was applied to the specimen. Water was then slowly flushed through the specimen from the bottom to flood the voids and remove as much air as possible prior to back pressure saturation. Water was allowed to flow through the specimen until no air bubbles were visually detected in the pipettes connected to the top of the specimen.

The back pressure saturation process was governed by the software program TEST. TEST indicates the increments to raise the cell and back pressure. Note that the maximum cell pressure available during the testing was only 106 psi (730.9 kPa). During testing, final B ( $B = \Delta u / \Delta \sigma_c$ ) values ranged from 0.80 to 0.92. Higher values of B indicate specimens with a higher degree of saturation. Various procedures were attempted to raise the final B value, including leaving the specimens to saturate for 24 hours and flooding the

specimens under vacuum; however, this practice did not prove to increase the B value significantly.

Hydraulic conductivity tests were then performed on the specimens. A pressure difference between the top and bottom of the specimen was applied to create a hydraulic gradient and the resulting flow rate through the specimen was measured. Results from the tests are discussed in Chapter 4.

The final step was the consolidated-drained triaxial shear test. Various effective confining pressures were applied to the specimens prior to shearing. After applying the confining pressure, the hydraulic pump was turned on via computer to allow the load piston to operate. Using the TEST software, all sensors were zeroed. The strain rate was set at 1% per minute for all tests, a rate sufficiently slow to allow full drainage of excess pore pressures. A final check of the cell pressure, back pressure, drain valves, axial strain reading, and load was conducted. Once again, using the TEST software to control the equipment, the test was initiated. Confining pressures were measured using a pressure transducer, while volumetric strains were recorded by monitoring the flow of pore water in or out of the specimen. Applied axial loads were measured using an electronic load cell, with axial deformations recorded with an LVDT. Shear test results are discussed in Chapter 5.

## Chapter 4: Results – Hydraulic Conductivity

### 4.1 HYDRAULIC CONDUCTIVITY ANALYSIS

Hydraulic conductivity values for all three materials were measured using a falling head - rising tail method in the triaxial testing apparatus. Hydraulic conductivity is calculated from the test results using the following equation (Gilbert 2001):

$$k_m = \frac{a_{in}a_{out}L}{(a_{in} + a_{out})A(t_2 - t_1)} \left( \ln \frac{h_1}{h_2} \right) \quad (4.1)$$

where  $k_m$  is the measured hydraulic conductivity (m/s) of the specimen, porous stones, and filter paper,  $a_{in}$  is the cross-sectional area of the influent reservoir ( $m^2$ ),  $a_{out}$  is the cross-sectional area of the effluent reservoir ( $m^2$ ),  $L$  is the length of the specimen along the flow path (m),  $A$  is the cross-sectional area of the specimen ( $m^2$ ),  $h_1$  is the difference in hydraulic head (m) across the specimen at time  $t_1$  (sec), and  $h_2$  is the difference in hydraulic head (m) across the specimen at time  $t_2$  (sec).

Because the test soils had a relatively high  $k_m$ , a correction accounting for the head loss in the porous stones and filter paper was applied. This correction assumes all the head loss in the system occurs in the stones and paper, although it actually applies to the whole system. This assumption allows the correction factor to be easily applied. After a value of  $k_m$  is calculated from Equation 4.1, it is corrected by the following equation:

$$k = \frac{Lk_{stone}}{(Lk_{stone}) + (L_{stone})(k_{stone} - k_m)} k_m \quad (4.2)$$

where  $k$  is the corrected hydraulic conductivity of the test material,  $L$  is the length of the specimen along the flow path,  $k_m$  is the value obtained from Equation 4.1,  $k_{stone}$  is the hydraulic conductivity of the two porous stones and filter paper, and  $L_{stone}$  is the length of the two porous stones and filter paper along the flow path. For these tests,  $L_{stone} = 0.525$  in (13.34 mm) and  $k_{stone} = 2.65 \times 10^{-4}$  cm/sec, which was measured in a test run with the same apparatus but without a soil specimen. In a typical case for RAP, a measured  $k_m = 1.53 \times 10^{-4}$  cm/sec resulted in a corrected  $k = 1.49 \times 10^{-4}$  cm/sec. Hence, this correction did not significantly alter the measured hydraulic conductivities.

Various factors affect the hydraulic conductivity of a soil specimen, including viscosity of the pore fluid, void ratio of the specimen, size and shape of the soil grains, degree of compaction, and degree of saturation. Specimens tested in this report were not 100% saturated. Thus, the presence of air bubbles impeded the flow of water and resulted in a lower  $k$ . Bovet et al. (1995) quantified the effect of partial saturation on hydraulic conductivity as follows:

$$k_{\text{non-saturated}} = k_{\text{saturated}} * (S_r)^5 \quad (4.3)$$

where  $k$  is the hydraulic conductivity and  $S_r$  is the degree of saturation of the specimen. Bovet et al. (1995) did not indicate what material they used in their tests. However, this relationship indicates that saturation has a significant effect

on hydraulic conductivity. In fact, based on this relationship, at  $S_r = 65\%$  the hydraulic conductivity is reduced by an order of magnitude, and at  $S_r = 40\%$  it is reduced two orders of magnitude, from the fully saturated sample.

#### **4.2 CONVENTIONAL FILL MATERIAL**

The measured hydraulic conductivity of the conventional fill material ranged from  $1.5 \times 10^{-4}$  to  $1.2 \times 10^{-3}$  cm/sec and averaged  $5.1 \times 10^{-4}$  cm/sec.

Hydraulic conductivity tests were run on specimen 1 at varying intervals during the back pressure saturation process to investigate the potential effect of saturation on the measured k values. As can be seen from Table 4.1, the hydraulic conductivity did not significantly change during the back pressure saturation process. For subsequent specimens, hydraulic conductivity was only measured after a final B value was obtained. Some hydraulic conductivity tests were performed twice on the same specimen, with the same B value, in order to verify the accuracy of the results. Table 4.2 shows the results of the hydraulic conductivity tests for the CFM.

Table 4.1. Effect of B Value on Hydraulic Conductivity of CFM Specimen 1.

B VALUE	0.40	0.47	0.80	0.80
K (CM/SEC)	$9.36 \times 10^{-4}$	$9.81 \times 10^{-4}$	$9.74 \times 10^{-4}$	$1.19 \times 10^{-3}$

Table 4.2. Conventional Fill Material Hydraulic Conductivity Data.

SPECIMEN #	1	2	3	4
ESTIMATED SATURATION (%)	-	93.4	-	94.4
FINAL B VALUE	0.80	0.84	0.80	0.90
K <sub>1</sub> (CM/SEC)	9.74*10 <sup>-4</sup>	1.49*10 <sup>-4</sup>	4.52*10 <sup>-4</sup>	3.68*10 <sup>-4</sup>
K <sub>2</sub> (CM/SEC)	1.19*10 <sup>-3</sup>	9.67*10 <sup>-5</sup>	4.74*10 <sup>-4</sup>	3.97*10 <sup>-4</sup>

Figure 4.1 shows the relationship between the hydraulic conductivity (average of two measured values per specimen) and B value, and the relationship between hydraulic conductivity and estimated saturation of the specimen. The value of the estimated saturation during hydraulic conductivity testing, S<sub>r</sub>, is calculated by the following equation:

$$S_r (\%) = (G_s * w_c) / e \quad (4.4)$$

where  $G_s$  is specific gravity,  $w_c$  is water content, and  $e$  is void ratio. To estimate the saturation level during the hydraulic conductivity tests, the water content and void ratio must be evaluated. Because triaxial tests were performed on the same specimens used for hydraulic conductivity tests, the water content, and consequently the saturation level of the hydraulic conductivity specimens, could not be directly measured. As a result, only the saturation level after triaxial testing was evaluated. The water content was measured after the triaxial testing from two small samples taken from within the center of the specimen. The void ratio after triaxial testing was estimated from the initial void ratio after compaction and volumetric strains measured during triaxial testing. These values of water content and void ratio may not be representative of the entire specimen, and, therefore, the saturation levels are only rough estimates.

The saturation is estimated for only two out of four CFM specimens because of a computer malfunction during specimen 3 and a failure to obtain the mass of specimen 1. Given this data, no clear relationship is observed between hydraulic conductivity and B or saturation in the CFM specimens.

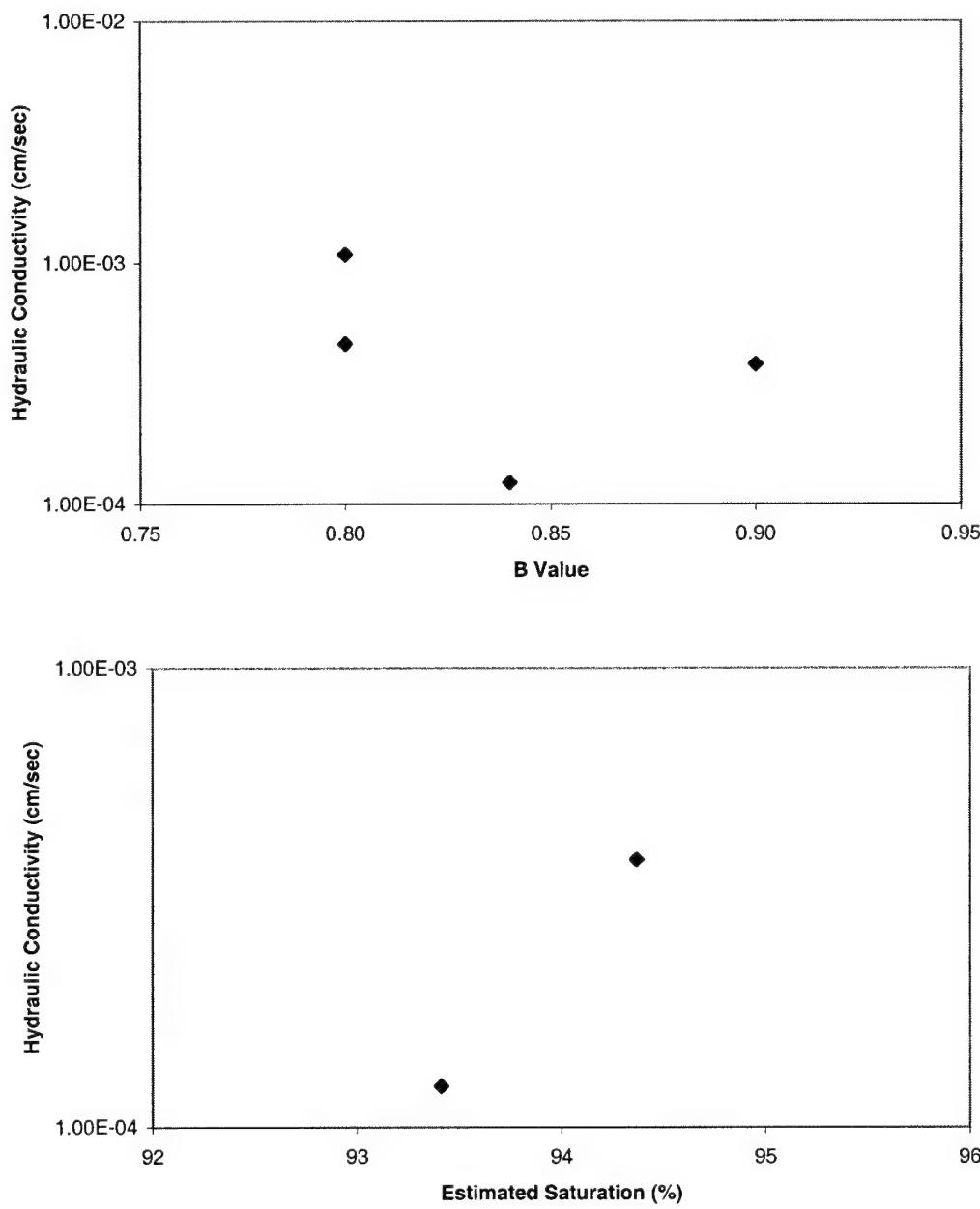


Figure 4.1. Effect of B Value and Estimated Saturation on Measured Values of CFM Hydraulic Conductivity.

### 4.3 CRUSHED CONCRETE

The measured hydraulic conductivity of the crushed concrete ranged from  $1.5 \times 10^{-6}$  to  $1.6 \times 10^{-5}$  cm/sec and averaged  $6.9 \times 10^{-6}$  cm/sec. Table 4.3 shows the results of the hydraulic conductivity tests for the CC.

Table 4.3. Crushed Concrete Hydraulic Conductivity Data.

SPECIMEN #	2	3	4	5
ESTIMATED SATURATION (%)	-	86.0	95.8	-
FINAL B VALUE	0.87	0.92	0.87	0.82
$K_1$ (CM/SEC)	$1.52 \times 10^{-6}$	$7.23 \times 10^{-6}$	$3.97 \times 10^{-6}$	$1.21 \times 10^{-5}$
$K_2$ (CM/SEC)	-	$8.60 \times 10^{-6}$	-	$1.60 \times 10^{-5}$

Figure 4.2 shows the relationship between the hydraulic conductivity (average of two measured values for specimen 3 and 5) and B value, and the relationship between hydraulic conductivity and estimated saturation of the specimen. The saturation is estimated for only two out of four CC specimens because one estimated value of saturation was over 100%, which is not reasonable, and a failure to obtain the final water content of specimen 2 prevented the estimation of saturation for that specimen. Again, the saturation values are very rough estimates, and Table 4.3 even shows that specimen 4, which has a lower B value than specimen 3, has a larger estimated saturation. Given this data,

Figure 4.2 indicates no clear relationship between hydraulic conductivity and B or saturation in the CC specimens.

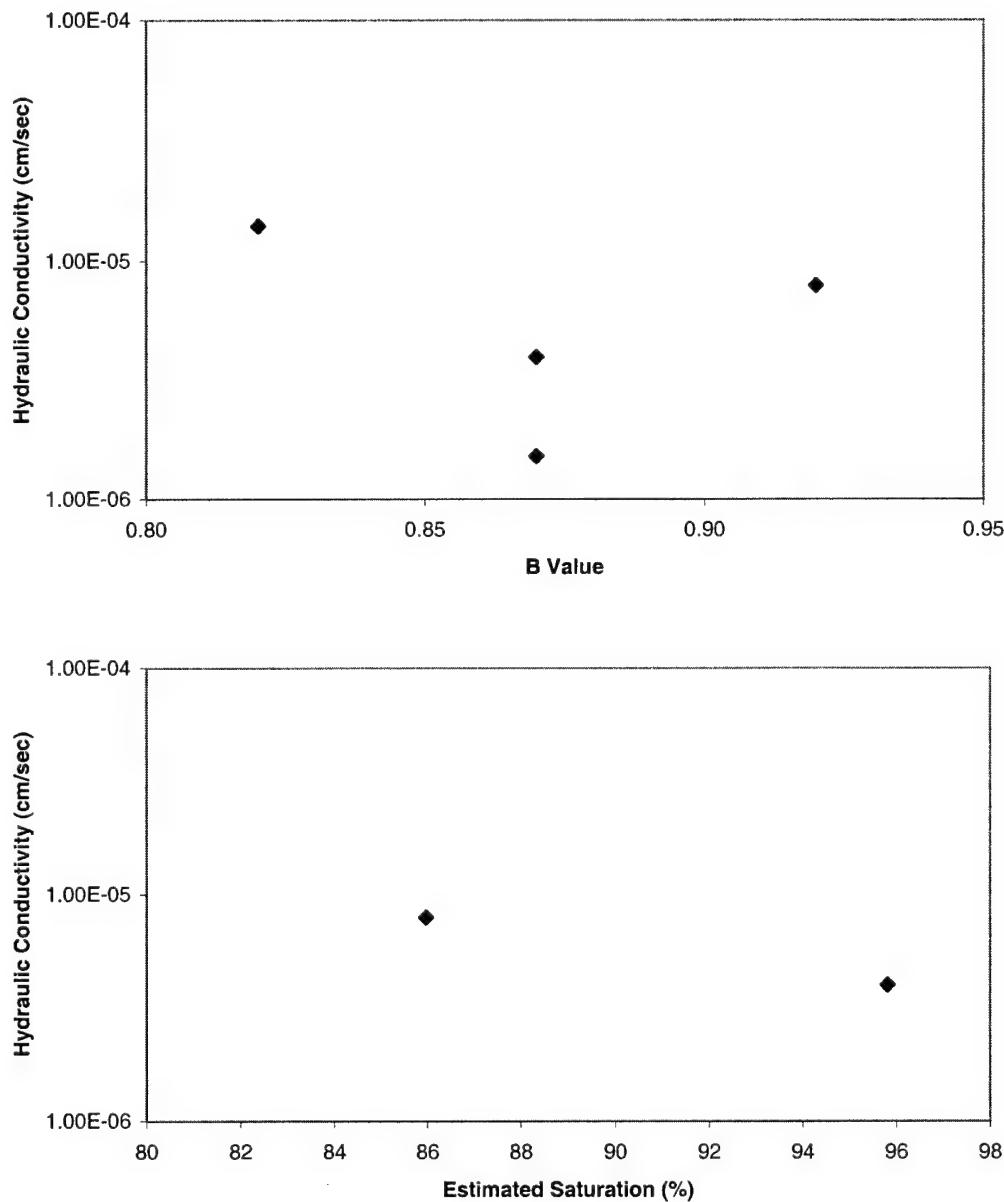


Figure 4.2. Effect of B Value and Estimated Saturation on Measured Values of CC Hydraulic Conductivity.

#### **4.3.1 Additional Observations on Crushed Concrete**

Some observations unique to the crushed concrete were noted during testing. The first involved the apparent cohesion of aggregate grains when mixed with water and left to dry. During the initial phases of testing, a CC specimen was prepared with an initial water content estimated to have been greater than 18%. The specimen was unstable due to its high water content and would not hold its shape when the mold was removed. The CC was placed in a bowl and allowed to dry overnight in the oven. Upon removal from the oven the following morning, the CC had a distinct, hardened crust. Additionally, the individual aggregate grains were noticeably conglomerated; they formed large chunks of cemented material that were visibly different from the original material. This oven-dried material was crushed by hand with metal scoops, mixed with water, and formed into a specimen. The specimen was tested for hydraulic conductivity and strength. The hydraulic conductivity measured was  $1.78 \times 10^{-4}$  cm/sec, two orders of magnitude larger than the average  $k$  obtained from the other CC specimens tested. The significant increase in hydraulic conductivity is believed to have resulted from having a coarser material following crushing of the hardened sample. This test was CC specimen 1. Due to the effects described above, the data from this test were not included in the evaluation of  $k$  for the CC material. The friction angle from this test was also  $2.5^\circ$  less than the average of the other CC specimens tested at the same effective confining stress.

To further study this phenomenon, the crushed concrete specimens were allowed to sit in the mold, prior to testing, for various times. Specimens 2 and 3

were tested immediately after compacting them in the mold. Specimen 4 was allowed to sit in the mold for 22 hours prior to testing. Finally, specimen 5 was allowed to sit in the mold for 118 hours prior to testing. Times were varied to investigate the potential difference in  $k$  values due to the possible hydration of the cement paste with increased set-up time. However, as indicated in Figure 4.3, the hydraulic conductivity values measured in these tests did not vary significantly with set up time.

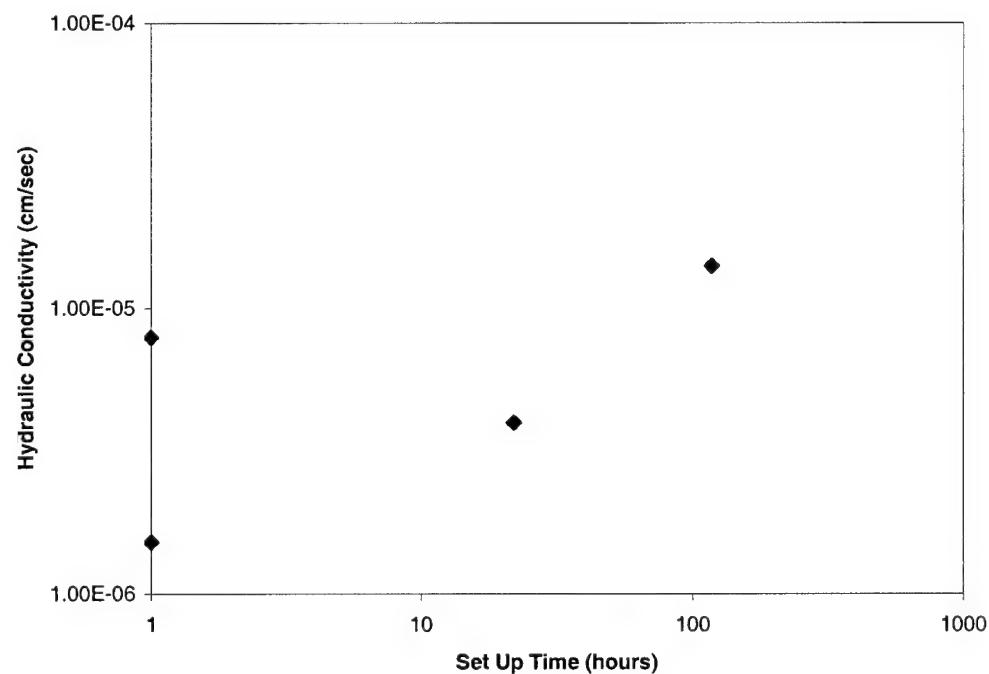


Figure 4.3. Effect of Set Up Time on Measured Hydraulic Conductivity of CC.

#### **4.4 RECYCLED ASPHALT PAVEMENT**

The measured hydraulic conductivity of the recycled asphalt ranged from  $1.4 \times 10^{-4}$  to  $3.6 \times 10^{-2}$  cm/sec and averaged  $7.3 \times 10^{-3}$  cm/sec. Table 4.4 shows the results of the hydraulic conductivity tests for the RAP.

Table 4.4. Recycled Asphalt Pavement Hydraulic Conductivity Data.

SPECIMEN #	1	2	3	4	5
ESTIMATED SATURATION (%)	72.5	-	91.1	90.4	94.7
FINAL B VALUE	0.86	0.82	0.80	0.82	0.83
$K_i$ (CM/SEC)	$2.47 \times 10^{-4}$	$9.31 \times 10^{-4}$	$2.57 \times 10^{-3}$	$1.68 \times 10^{-3}$	$3.57 \times 10^{-2}$
$K_s$ (CM/SEC)	$1.41 \times 10^{-4}$	-	$2.22 \times 10^{-3}$	$1.42 \times 10^{-3}$	$2.75 \times 10^{-2}$

Figure 4.4 shows the relationship between the hydraulic conductivity (average of two measured values per specimen except for specimen 2) and B value, and the relationship between hydraulic conductivity and estimated saturation of the specimen. The saturation is estimated for only four out of five RAP specimens because one estimated value of saturation was over 100%, which is not reasonable. As observed from the CC, the estimated values of saturation do not correlate with the measured B values, indicating that the saturation values

may be erroneous. Given this data, Figure 4.4 indicates no clear relationship between hydraulic conductivity and B or saturation in the RAP specimens.

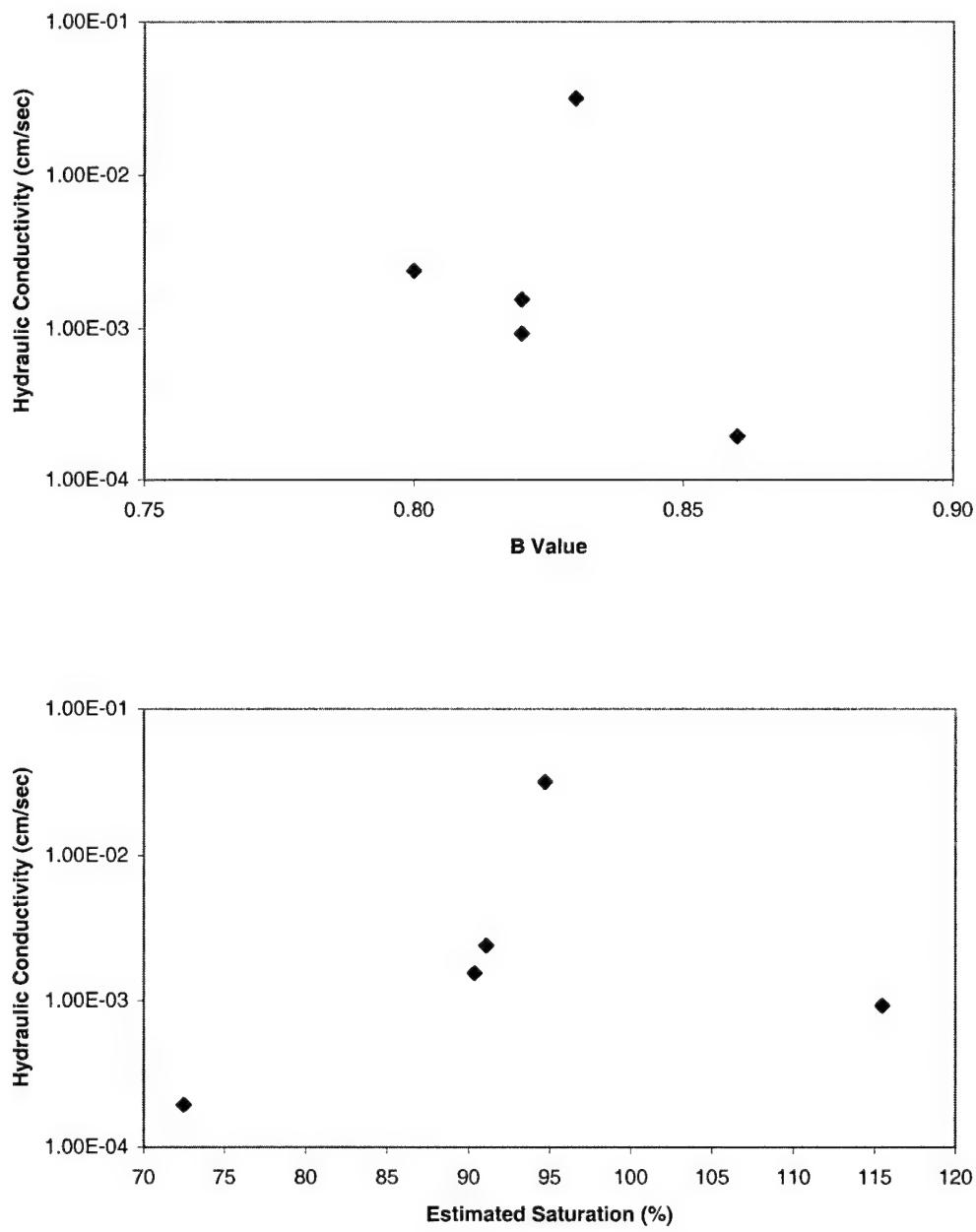


Figure 4.4. Effect of B Value and Estimated Saturation on Measured Values of RAP Hydraulic Conductivity.

## Chapter 5: Results – Shear Strength

### 5.1 SHEAR STRENGTH ANALYSIS

Results from consolidated-drained triaxial compression tests were plotted in terms of deviator stress ( $\sigma'_1 - \sigma'_3$ ) and volumetric strain ( $\epsilon_v$ ) versus axial strain ( $\epsilon_a$ ). The deviator stress was computed by dividing the applied axial load (F) by the current cross-sectional area of the specimen ( $A_c$ ), computed assuming a right circular cylinder area correction. In equation form, where  $A_o$  is the original cross-sectional area:

$$(\sigma'_1 - \sigma'_3) = \frac{F}{A_c} = \frac{F}{A_o} \frac{(1 - \epsilon_v)}{(1 - \epsilon_a)} \quad (5.1)$$

Failure was defined as the peak stress difference in each test.

The effective stress friction angle ( $\phi'$ ) and cohesion intercept ( $c'$ ) were evaluated by the following method. Values of  $Q$  ( $[\sigma'_1 - \sigma'_3]/2$ ) and  $P$  ( $[\sigma'_1 + \sigma'_3]/2$ ) at failure were calculated for each test and a straight line was fit through a plot of the data. The slope of this line ( $m$ ) is related to  $\phi'$ , and the intercept ( $b$ ) is related to  $c'$ , as:

$$\phi' = \sin^{-1} (m) \quad (5.2)$$

$$c' = b / \cos \phi' \quad (5.3)$$

## **5.2 CONVENTIONAL FILL MATERIAL**

The results from the four tests on CFM are summarized in Table 5.1. Tests were performed at effective confining pressures between 12 and 40 psi (82.7 and 275.8 kPa). All tests exhibited strain softening behavior, reaching a maximum deviator stress at approximately two percent strain and then declining to a residual value (Figure 5.1). This response is typical for a dilative material in drained shear. Only three data sets are shown on Figure 5.1 because the stress-strain data were inadvertently not collected for specimen 3. Additionally, all specimens displayed a distinct failure plane through the specimen during shear. The friction angle fit to all four of the tests is  $54.7^\circ$  and the cohesion intercept is zero. Strength parameters from three of the four tests are graphically depicted in Figure 5.2.

Table 5.1. Conventional Fill Material Strength Data.

SPECIMEN #	1	2	3	4
TOTAL UNIT WEIGHT (LBS/FT <sup>3</sup> )	-	139.7	139.2	140.1
INITIAL SATURATION(%)	-	76.4	73.4	80.3
INITIAL WATER CONTENT(%)	5.5	8.2	7.9	8.7
DRY UNIT WEIGHT (LBS/FT <sup>3</sup> )	-	129.1	129.0	128.9
VOID RATIO	-	0.28	0.29	0.29
CELL PRESSURE (PSI)	103	103	106	105
BACK PRESSURE (PSI)	80	91	72	65
EFFECTIVE CONFINING STRESS (PSI)	23	12	34	40
MAX DEVIATOR STRESS (PSI)	227.4	114.9	316.1	341.7
ϕ' (DEG)	56.3	55.8	55.4	54.1

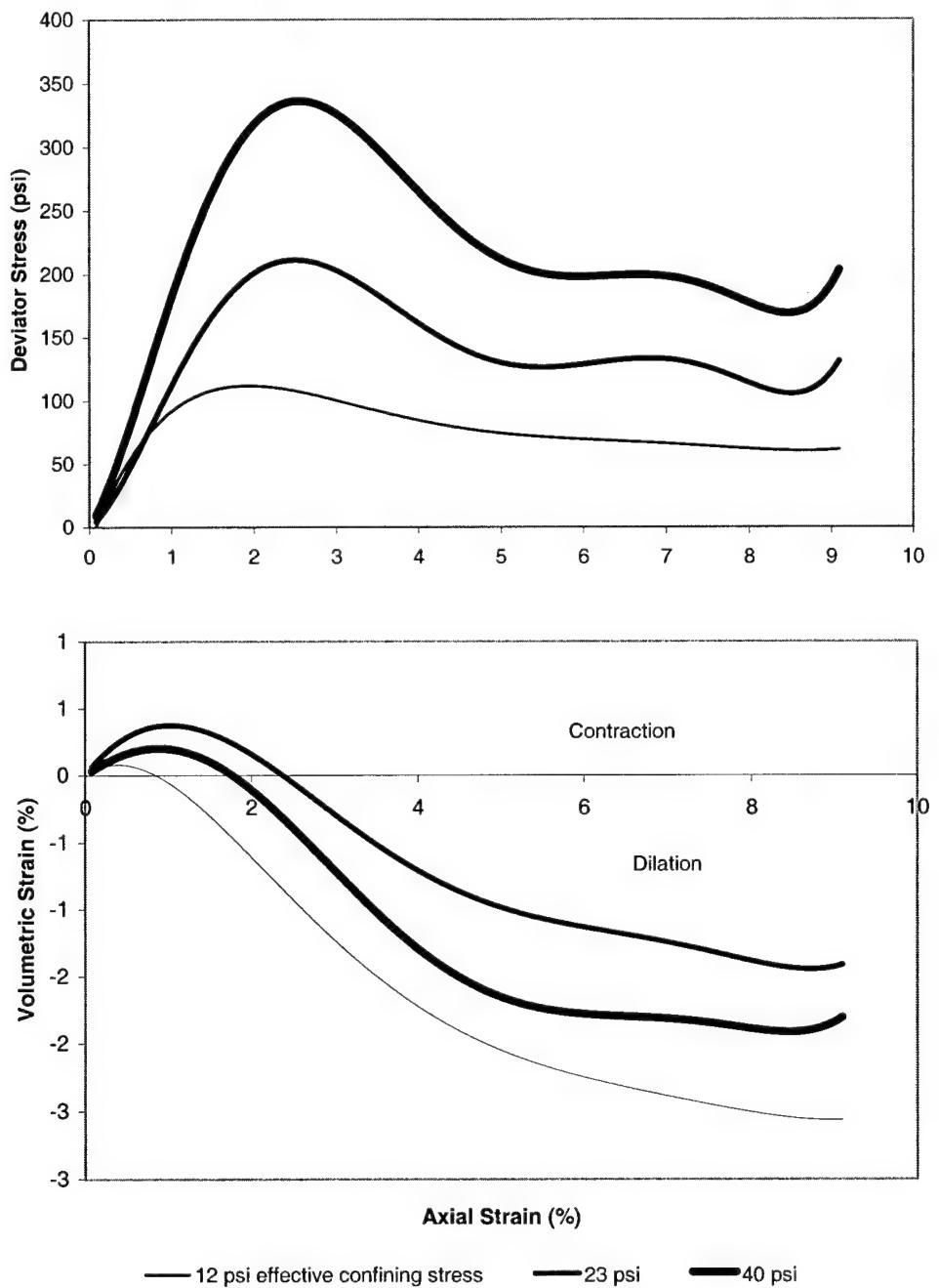


Figure 5.1. Conventional Fill Material Triaxial Test Results.

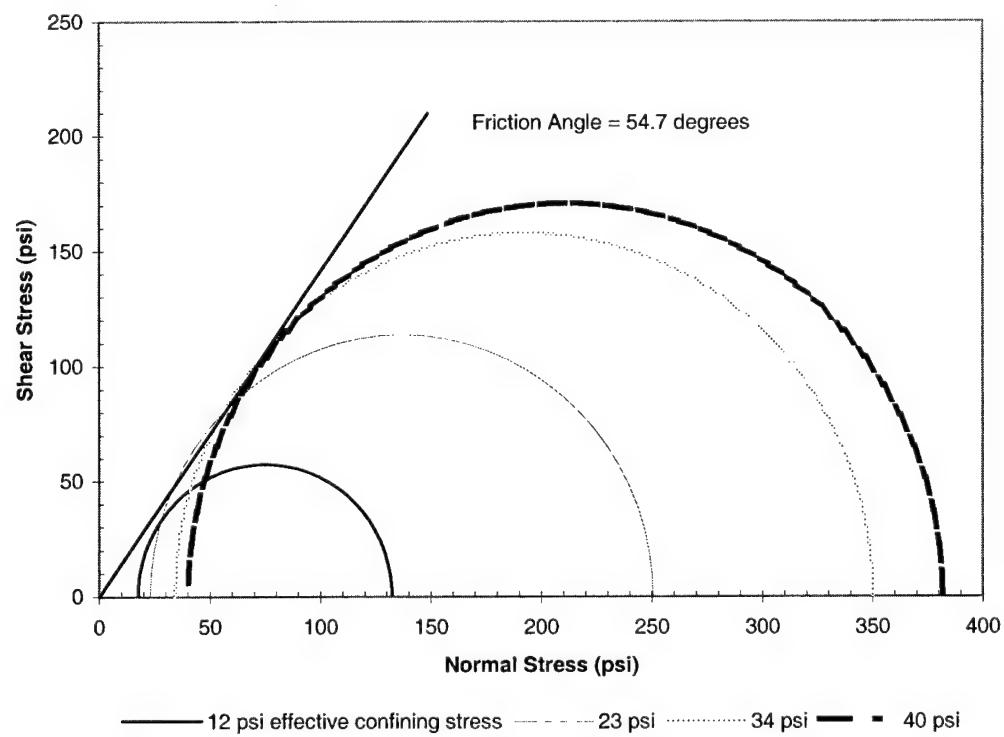
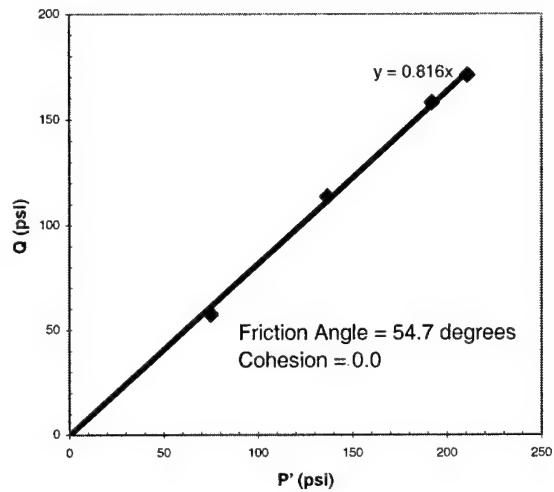


Figure 5.2. Conventional Fill Material Friction Angle and Mohr's Circles.

### **5.3 CRUSHED CONCRETE**

The results from the four tests on CC are summarized in Table 5.2. Tests were performed at effective confining pressures between 12 and 37 psi (82.7 and 255.1 kPa). All tests exhibited strain softening behavior, reaching a maximum deviator stress at approximately two percent strain and then declining to a residual value (Figure 5.3). Additionally, all specimens displayed a distinct failure plane through the specimen during shear (Figure 5.4). This response is typical for a dilative material in drained shear. The friction angle fit to all of the four tests is  $54.3^\circ$  and the cohesion intercept is zero. Strength parameters from the four tests are graphically depicted in Figure 5.5.

Table 5.2. Crushed Concrete Strength Data.

SPECIMEN #	2	3	4	5
TOTAL UNIT WEIGHT (LBS/FT <sup>3</sup> )	135.0	135.0	133.8	135.2
INITIAL SATURATION (%)	96.4	95.2	92.0	95.5
INITIAL WATER CONTENT (%)	14.0	13.7	13.7	13.6
DRY UNIT WEIGHT (LBS/FT <sup>3</sup> )	118.4	118.7	117.7	119.0
VOID RATIO	0.38	0.38	0.39	0.37
CELL PRESSURE (PSI)	105	100	103	103
BACK PRESSURE (PSI)	68	88	80	80
EFFECTIVE CONFINING STRESS (PSI)	37	12	23	23
MAX DEVIATOR STRESS (PSI)	333.0	106.9	190.7	204.1
φ' (DEG)	54.9	54.8	53.7	54.7

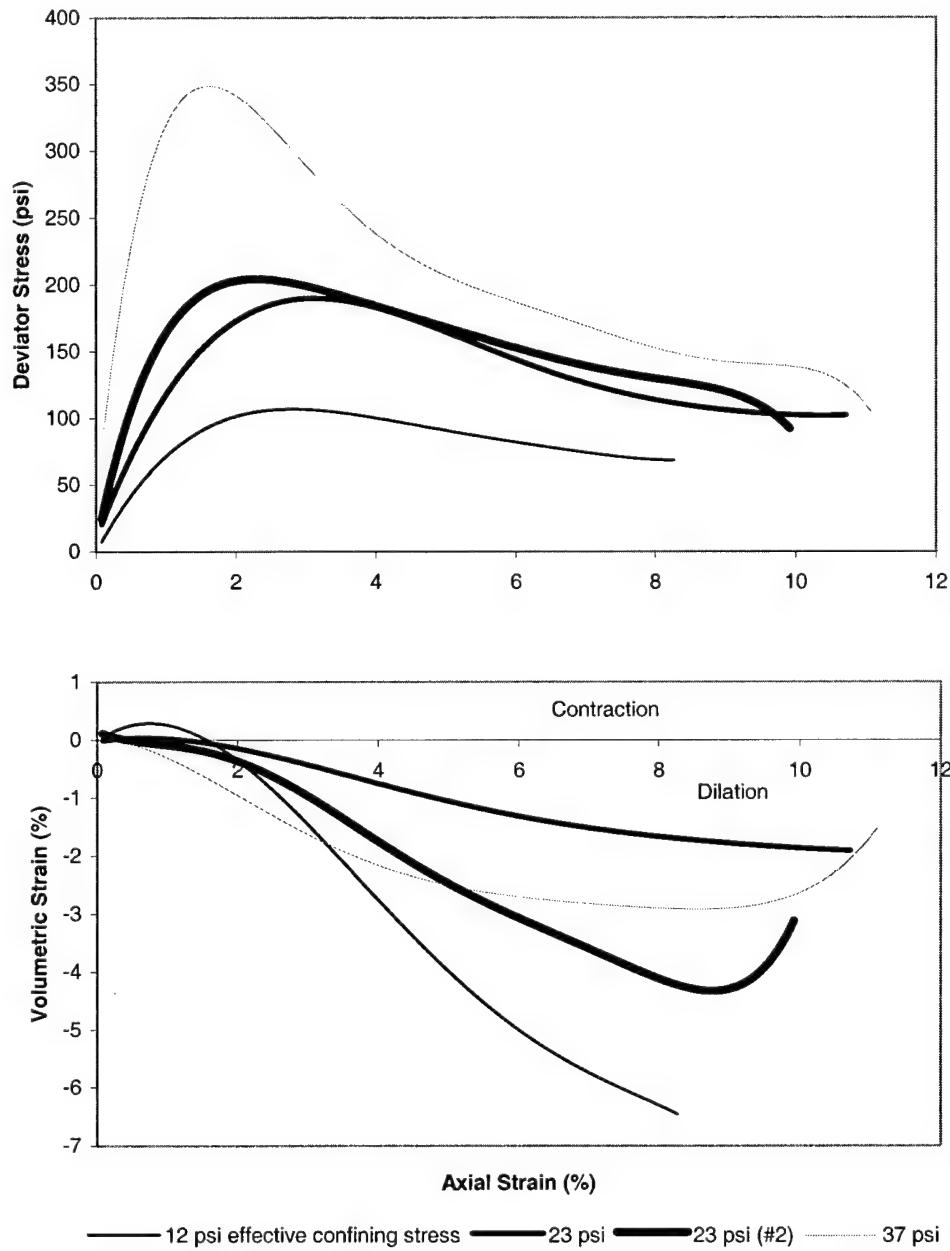


Figure 5.3. Crushed Concrete Triaxial Test Results.

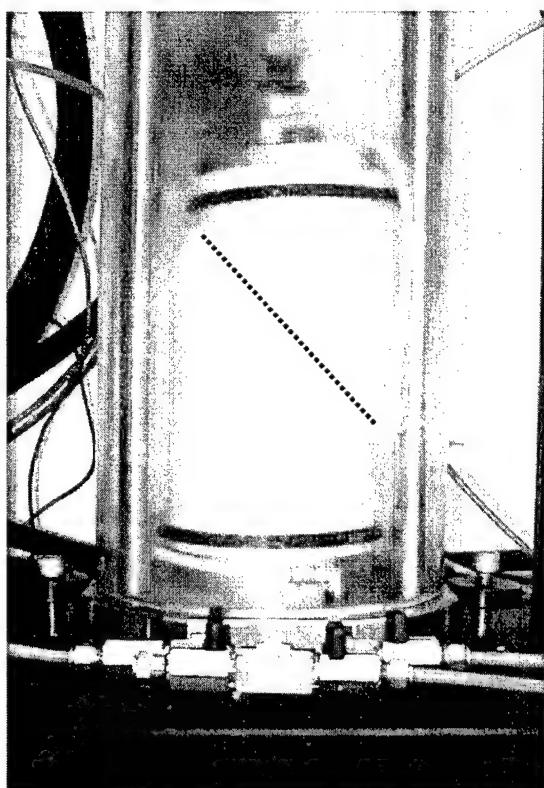


Figure 5.4. Crushed Concrete with Visible Failure Plane Through the Specimen.

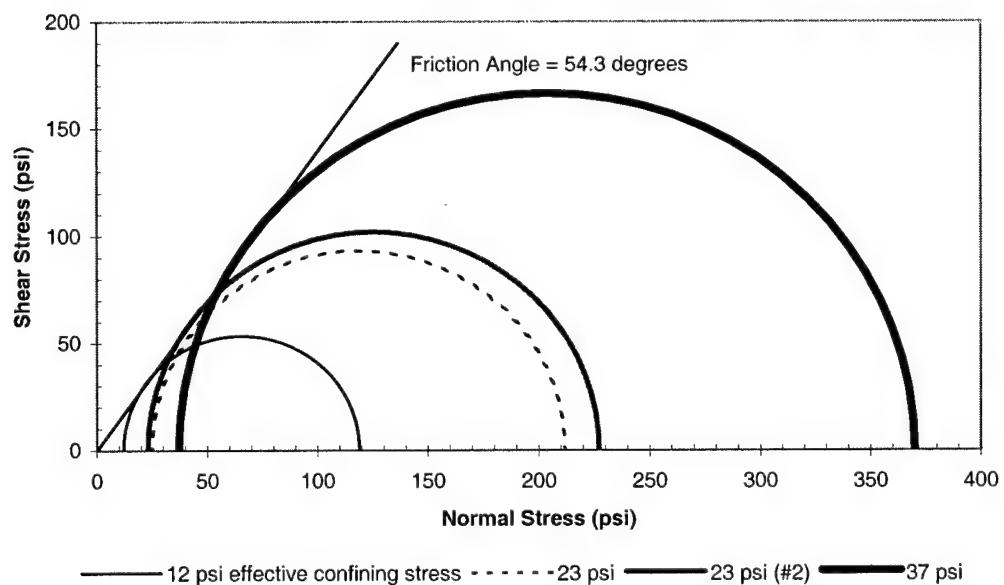
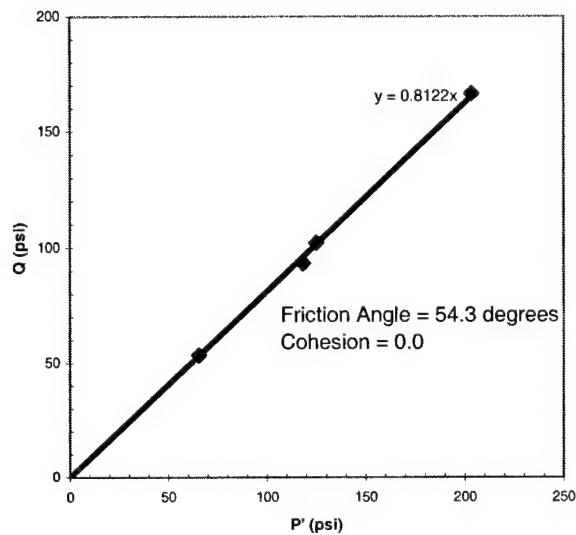


Figure 5.5. Crushed Concrete Friction Angle and Mohr's Circles.

## **5.4 RECYCLED ASPHALT PAVEMENT**

The results from the five tests on RAP are summarized in Table 5.3. Tests were performed at effective confining pressures between 12 and 45 psi (82.7 and 310.1 kPa). All tests exhibited strain hardening behavior with the deviator stress continuing to rise or remaining steady throughout the duration of the test (Figure 5.6). Volumetric strains ranged from dilative to contractive, in stark contrast to the large dilative volumetric strains observed in the CFM and CC (Figures 5.1 and 5.3). Additionally, no specimens displayed a distinct failure plane during shear. Rather, the specimens simply compressed vertically and exhibited a slight radial bulge near the center as the axial load was applied (Figure 5.7). This behavior is typical of contractive material in drained shear. It is surprising that compacted specimens would exhibit contractive behavior. This may be a result of the bitumen in the RAP. The friction angle resulting from the five tests is  $39.0^\circ$ . A cohesion value of 8.0 psi (55.1 kPa) was calculated using Equation 5.2. Strength parameters from the four tests are graphically depicted in Figure 5.8.

Table 5.3. Recycled Asphalt Pavement Strength Data.

SPECIMEN #	1	2	3	4	5
TOTAL UNIT WEIGHT (LBS/FT <sup>3</sup> )	124.2	124.1	124.9	124.1	123.8
INITIAL SATURATION (%)	68.6	54.7	48.3	46.4	47.3
INITIAL WATER CONTENT (%)	6.7	4.9	3.9	3.9	4.1
DRY UNIT WEIGHT (LBS/FT <sup>3</sup> )	116.4	118.3	120.2	119.4	118.9
VOID RATIO	0.22	0.20	0.18	0.19	0.20
CELL PRESSURE (PSI)	103	103	100	103	103
BACK PRESSURE (PSI)	80	66	88	58	86
EFFECTIVE CONFINING STRESS (PSI)	23	37	12	45	17
MAX DEVIATOR STRESS (PSI)	107.3	166.7	74.7	181.9	93.2
φ' (DEG)	44.4	43.8	49.2	42.0	47.1

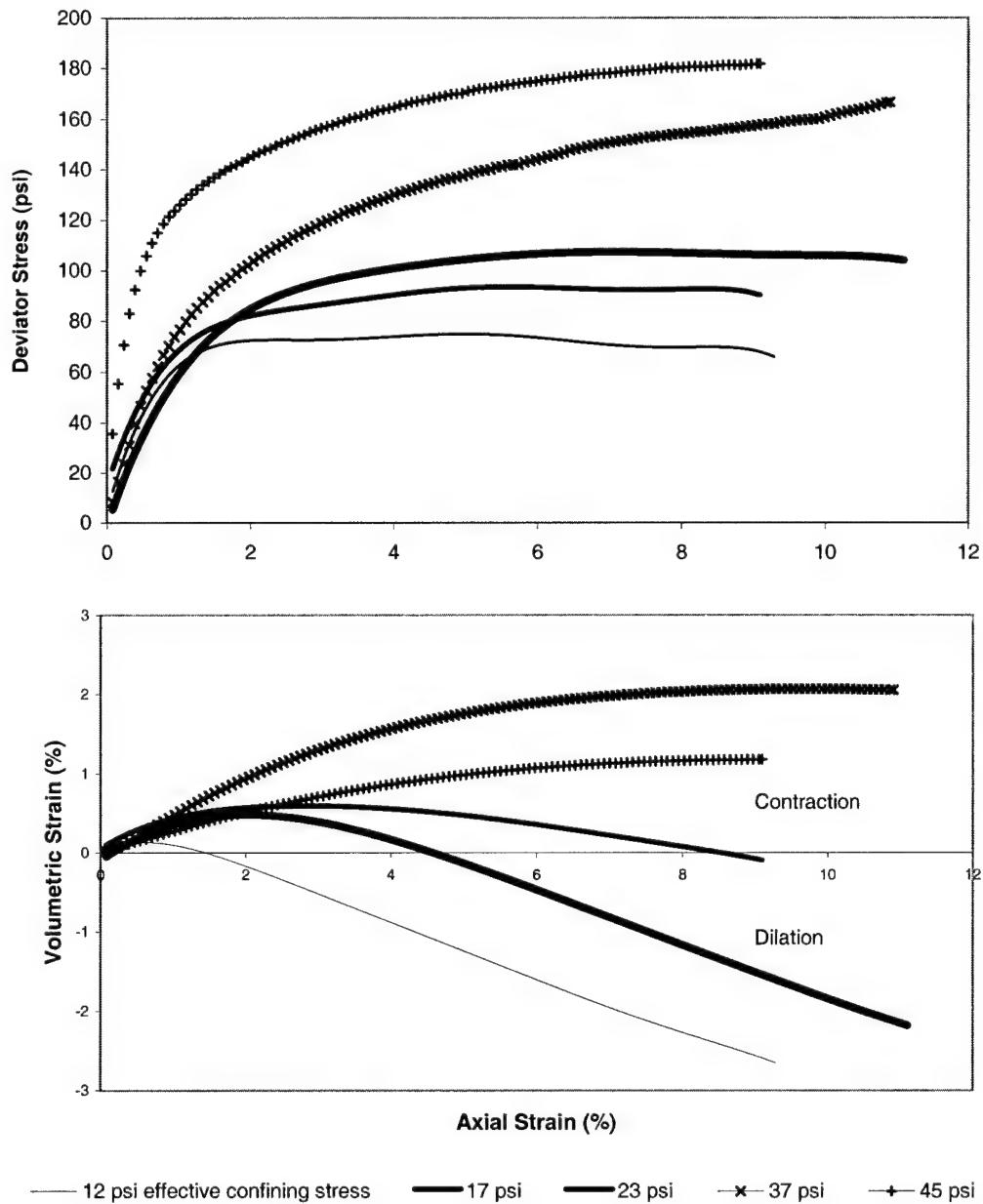


Figure 5.6. Recycled Asphalt Pavement Triaxial Test Results.

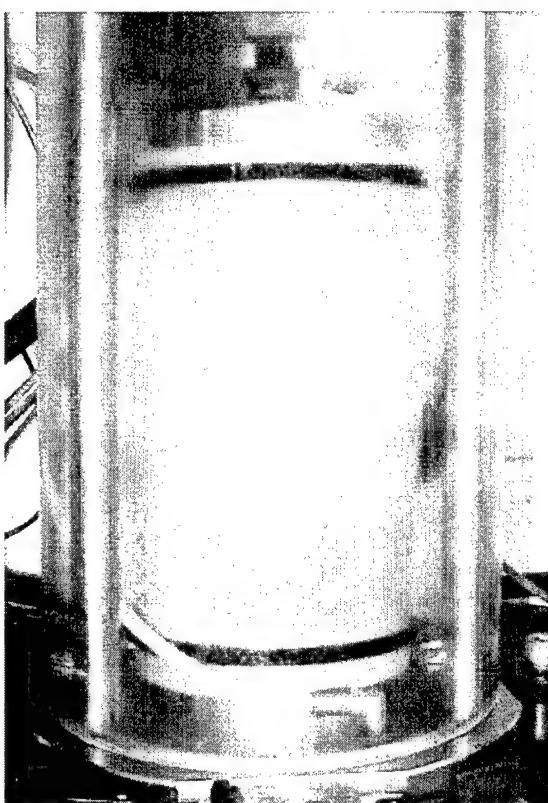


Figure 5.7. Sheared Recycled Asphalt Pavement with No Visible Failure Plane.

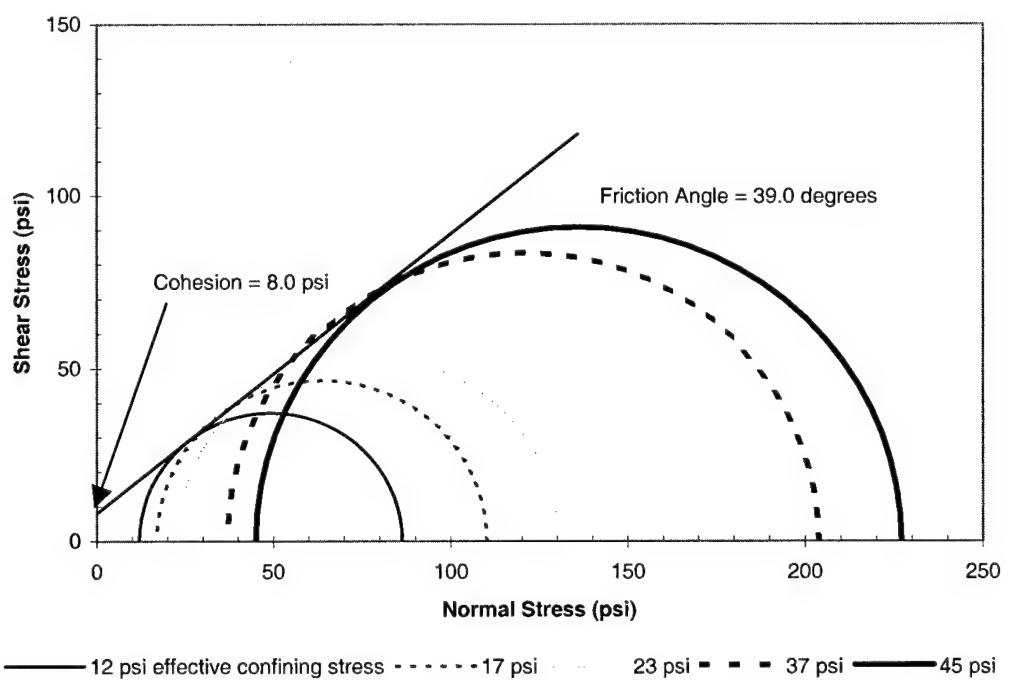
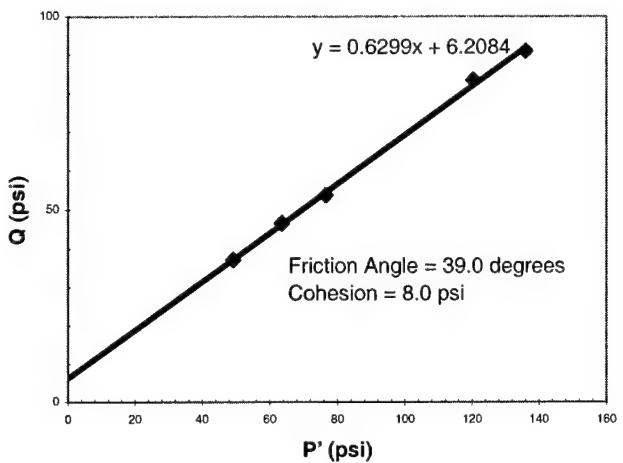


Figure 5.8. Recycled Asphalt Pavement Friction Angle and Mohr's Circles.

## Chapter 6: Conclusion

### 6.1 COMPARISON OF MEASURED VALUES

Table 6.1 provides the hydraulic conductivity and shear strength test results for all three materials tested.

Table 6.1. Overall Test Results.

	CFM	CC	RAP
EFFECTIVE CONFINING PRESSURE RANGE (PSI)	12 – 40	12 – 37	12 – 45
EFFECTIVE STRESS FRICTION ANGLE, $\phi'$	55°	54°	39°
EFFECTIVE COHESION, $c'$ (PSI)	0	0	8
HYDRAULIC CONDUCTIVITY, $K$ (CM/SEC)	$5 \times 10^{-4}$	$7 \times 10^{-6}$	$7 \times 10^{-3}$

### 6.2 RECOMMENDATIONS FOR CRUSHED CONCRETE

Compared to the conventional fill material, the crushed concrete has similar shear strength parameters,  $c'$  and  $\phi'$ . Both the CFM and CC are highly angular materials with relatively high drained friction angles at confining pressures up to 40 psi, although lower friction angles can be expected at higher

confining pressures. Also, because the tests reported here were conducted on specimens with all particles larger than 0.63 inch removed, the strength of the composite material may be somewhat lower, as the biggest particles of this crushed material may be more fractured. While additional tests are warranted, the shear strength of CC appears to be adequate for MSE wall backfills.

The drainage properties of the backfill material for retaining walls are also very important. If a poorly draining material is placed behind an earth retaining structure, water may collect behind the wall. This could cause excessive water pressures to build up that could lead to large wall deformations or collapse.

As discussed previously, a poorly draining material may be defined as a material with a hydraulic conductivity less than  $1.0 \times 10^{-4}$  cm/sec (Carter and Bentley 1991). Test results from this study indicate an average value of  $7 \times 10^{-6}$  cm/sec for crushed concrete. However, only four specimens were tested and all were partially saturated. The partial saturation of the specimens would be expected to yield a lower measured hydraulic conductivity. It is important to note that the Michigan Department of Transportation does not allow the use of CC with particles smaller than 4.75 mm in drainage layers.

Therefore, it is recommended that further studies be performed concerning the drainage properties of CC. This information is needed before a conclusion on its potential use as a backfill material can be reached. Future studies should focus on the variables associated with specimen preparation (i.e., gradation size, set-up time prior to testing, and compaction efforts) and how they affect hydraulic conductivity. Adequate hydraulic conductivity might be obtained by removing

some of the finer particles in the CC samples. Additionally, due to the anticipated range of values expected from the tests, one should perform them using a constant head apparatus that allows for slow flow rates.

### **6.3 RECOMMENDATIONS FOR RECYCLED ASPHALT PAVEMENT**

Although the measured strength of the recycled asphalt pavement was lower than that for the conventional fill material, the RAP tested appears to exhibit adequate strength properties to be used as a backfill material. Additionally, the hydraulic conductivity of the RAP tested was larger than that of the conventional fill material.

The lower friction angle for RAP, compared to the CFM, may result from the more rounded aggregates present in the RAP. Note also that the RAP exhibited a drained strength envelope with  $c' = 8$  psi. This apparent cohesion likely results from the residual bitumen bonding the particles together. However, it is not clear if this component of shear strength can be relied upon for design. Additional studies are needed to assess the stability of this material, particularly to determine if RAP is susceptible to creep failure at large shear stress levels. As with the crushed concrete, additional studies are needed to evaluate if removing particles larger than 0.63 inch from these test specimens had a significant effect on the measured strength.

As discussed in Section 2.3, at least nine states have used RAP as an embankment construction material. More recently, at least five states have used RAP directly as backfill material. The Federal Highway Administration has noted

the performance of RAP in these projects has ranged from satisfactory to good. One cautionary measure is noted during the compaction phase of construction; visual inspections on a continuous basis should be performed to ensure that the specified degree of compaction is achieved, or that there is no movement under the action of compaction equipment. Other recommendations regarding the use of RAP as an embankment material are discussed in Section 2.3.

Based on the results of this study, RAP appears to have adequate strength and hydraulic conductivity to be considered for use in mechanically stabilized earth walls.

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## **Vita**

Patrick James Carley was born in Silver Spring, Maryland on April 15, 1972, the son of William Patrick Carley and the former Busra Mekvishai. Patrick was raised in Burke, Virginia, part of the greater Washington, D.C. area. In 1990, he entered Texas A&M University in College Station, Texas. During his undergraduate education, he completed the cooperative education program by working with the Illinois Department of Transportation for 13 months. He received his Bachelor of Science in Civil Engineering from Texas A&M University in May 1995. Two weeks later, he married the former Stephanie Anne Pharr. Patrick entered the United States Air Force in June 1995. His assignments included three years at Bolling Air Force Base, Washington, D.C., and two-years at Lajes Field, Azores, Portugal. In August 2000, the Air Force sent him to graduate school to obtain his master's degree. Patrick and Stephanie were blessed with a healthy baby boy, Joshua Seamus Carley, in September 2000. Patrick is currently a Captain in the United States Air Force and upon graduation will be assigned to Minot Air Force Base, North Dakota. Gig 'Em Aggies!

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